Flexible Stadium Design in the Context of Olympics and Post-Olympics Usage

By

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Abstract

The design of the London Olympic Stadium for the 2012 Summer Olympic Games represented a shift in traditional stadium design for major sport events on the scale of the Olympics or World Cup. Emphasising design with a focus towards post-Olympics usage, the London Olympic Stadium through features like a demountable second seating tier, reclaimed steel elements, and structurally isolated façade, set a strong precedent for flexible Olympics stadium construction. The goal of this thesis is to quantitatively explore options to push the boundaries of flexible stadium design, easing the renovation process required to transition stadiums from Olympics to post-Olympics usage.

Through case study design examples, this thesis explores the effect bolted rather than welded connections can have on the design of stadium grandstands. Evaluated for both strength and serviceability, this thesis applies work demonstrating the pros and cons of bolted connections in traditional braced frame structures to stadium grandstands. Finally, this thesis explores the opportunity events like the Olympics provide to perform a probabilistic performance based design on an elliptical roof truss system. Given current building codes specify loads intended for use in the design of permanent structures, this thesis breaks down building code methodology in an attempt to determine loads more appropriate for use in the design of buildings with intended life spans on the order of an Olympic cycle (four years). Looking specifically at a stadium structural system typically controlled by wind and snow loads, this thesis attempts to quantify the material savings possible when designing a structure using performance rather than code based design.

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1 Introduction

In an interview describing the NFL's Seattle Seahawks' come from behind victory over the Green Bay Packers in the 2015 NFC Championship game, NFL writer and Grantland contributor Robert Mayes described why we enjoy football and sports in general. "I think that's why we care so much… and that's why we find [sports] so interesting. Because the escapism part [of our passion for sports] isn't just imagining ourselves in [the athletes'] shoes. It's the moments where we know we can't" (Mayes and Barnwell). Inspired by the amazing acts of athleticism, skill and coordination beyond that of the average human being, sports allow us to feel a sense of awe and pride in the athletes we watch, reveling in the drama sporting events interject into our everyday lives.

Stadiums, in locations around the world, serve as the backdrop for these dramas. From the extruded aluminum bleachers that line the sidelines of high school football and baseball fields across America, to the architectural masterpieces that host World Cup Finals and Olympics Opening Ceremonies, stadiums throughout history have been the stages on which athletes around the world compete, and spectators gather to observe often amazing feats.

Within the context of major athletics competitions, specifically events on the scale of the Olympics or World Cup, much has been written on the "white elephant" creating nature of these games. Like a rude party guest that leaves without helping to clean up, events like the Olympics or World Cup have a history of requiring massive investments from host cities in sporting infrastructure (i.e. stadiums and support facilities), without providing any significant guidance for what to do with this infrastructure following the event.

Judging by the most recent Summer Olympics (London, 2012), it would appear a corner may have been turned, however, in the history of short term planning for major sporting competitions. In an attempt to help remedy the trend of short term thinking in major event planning, the primary purpose of this thesis will be to examine whether there is more the field of structural engineering can contribute to help further the push for long term planning in stadium design, evaluating various structural forms as they relate structural efficiency to long term economy.

2 Background

During the most recent Summer Olympic Games in London, the International Olympic Committee (IOC) estimates almost 900 million people tuned in to watch the opening ceremonies world-wide (Ormsby, 2012). To put that number in perspective, of the most recently estimated 1.4 billion households across the world who currently own at least one television (Butts, 2013), it is estimated that upwards of 60% of those households watched at least some portion of the London Olympics Opening Ceremonies. With these statistics in mind, it is undeniable the Olympic Games has grown beyond a simple athletic competition, and transformed into truly a global event with reach in households the world over; and as the games have grown, so too have the venues they have been held in.

2.1 The Modern Olympic Games

While the Olympic Games traces its origins back to ancient Greece, the first modern Games in the form most similar to the ones we know today were held in Athens, Greece in 1896. The brainchild of Barron Pierre de Coubertin, the first modern Olympiad was held in the ancient Panathenaic Stadium which had been restored for the occasion. Maintaining a number of similarities with the ancient games including several of the original sporting events, one large overarching theme of the games throughout its history has been a motivation towards improving international relations through sport; a concept dubbed the Olympic Truce. Drawing inspiration from the games of the ancient Greeks, the Olympic Truce "is the subject of a United Nations resolution calling for a halt to hostilities during the period of the Games and the search for means of peaceful resolution in areas of tension" (The Olympic Museum Educational and Cultural Services, 2012).

In its modern incarnation, the founders of the Olympics and subsequent Olympic organizers have made efforts to transform the Games into a global event, making changes in schedule and format, underlying themes, and participants. At its core, the ancient Olympics had roots in religion, honoring the gods through sport. In updating the Games into a global event, the modern Olympics were founded as a secular event with an emphasis on making strides towards creating a more peaceful world. Along the lines of a global event, in his original proposal for the Olympic Games, Coubertin intended for the Games to be held in cities around the world.

From 245 participants representing 14 countries at the Athens Olympics in 1896, the Olympics as an event has grown in a number of dimensions including duration, competitors and countries represented. The first Winter Olympics was held in 1924 in Chamonix, France with 258 athletes competing representing 16 countries. In the most recent Winter Games (Sochi 2014), 2,873 athletes participated representing 98 countries in 15 different disciplines. To date, between the Summer and Winter Olympics, the Olympic Ceremonies have been held in 42 unique cities in 23 countries across 5 continents (The Olympic Museum Educational and Cultural Services, 2012).

	Summer Olympics		Winter Olympics	
	First	Last	First	Last
Year:	1896	2012	1924	2014
Site:	Athens, Greece	London, UK	Chamonix, France	Sochi, Russia
Participants:	245	10,700	258	2,873
Countries:	14	204	16	98
Duration:	10 days	16 days	11 days	16 days

Table 1. First and last Olympics statistics

Beyond the expansion of the Olympics as a global completion, one of the most significant departures the modern Games have adopted in comparison to the Games of antiquity has been opening up the pool of competitors beyond exclusively male, Greek citizens. Although the process of women inclusion in the Games has not always been the smoothest, female athletics in the Olympics has grown steadily from the first female competitions in the 1900 Summer Olympics in Paris. In the earliest Games, female athletes were limited to compete in tennis and golf. It took until the London Olympic Games in 2012 with the introduction of women's boxing, that women could compete in the same number of sports men could compete in.

When founding the modern Olympics, Barron Pierre de Coubertin stated publicly his intention that the games be limited to only amateur athletes. Giving rise to such iconic sports moments in American sports history as the 1980 United States Men's Ice Hockey team's "Miracle on Ice" victory over the Soviet Union, this history of strict amateurism had been one of the core tenants of the Olympic Games, until 1984 when the policy was abolished. Since the 1984 Games in Los Angeles, professional athletes have been able to compete in all competitions.

2.2 Olympic Stadiums throughout History

2.2.1 Athens 1896: The First Modern Olympic Stadium

The site of the first modern Olympics, the Panathenaic Stadium in Athens has a long history of use as both an athletics arena, and public gathering space. The site traces its history back to roughly 566 BC where athletic competitions involving nude male athletes took place in an event known as the Panathenaia festival. Although the stadium has taken various forms throughout its history, the structure reconstructed for the 1896 Olympics draws its influences from the stadium built between AD 139 and 144 by the Roman Herodes, designed as a horse-shoe shape with a vaulted passageway constructed under the grandstand. Built entirely out of marble, the single tiered Panathenaic Stadium that stands today can seat roughly 45,000 people, and was reused as the site of the archery event for the 2004 Athens Olympic Games.

2.2.2 Amsterdam 1928: The Olympic Torch

The 1928 Olympic Games held at the Olympic Stadium in Amsterdam, Netherlands, was the first of the modern games to incorporate the lighting of the now iconic Olympic Torch. Constructed in 1927, the Olympic Stadium was designed to host a number of different events including soccer, track and field, and cycling among others. While it is not uncommon for many of today's stadiums to include both an athletics track and a soccer pitch, the inclusion of the additional 9 meter wide cycling ring surrounding the athletics track decreased the stadium's capacity by roughly 20% (Olympisch Stadion Amsterdam, 2011). This multi-use design criteria may have helped improved the stadium's longevity in the long-term however as it has been in continuous use by various permanent tenants since the games.

Figure 1. The Amsterdam Olympic Stadium during the Opening Ceremonies of the 1928 Games

2.2.3 Berlin 1936: Politics in the Olympics

Built as the main site of the 1936 Berlin Olympics, the Berlin Olympic Stadium (also known as the Olympiastadion) was constructed between 1934 and 1936 and was built as a part of a larger athletics complex that included a large parade ground known as the Maifeld and Waldbühne amphitheater. The original design for the stadium called for the lower portion of the bowl be built roughly 12 meters below grade, with the upper portion of the stadium supported by poured concrete columns around the stadium perimeter. Perhaps the stadium's most architecturally distinct feature is the gap in the stadium's elliptical form with a direct sightline towards a view of the bell tower at the western edge of the site. Although unfortunately best known for the propaganda images taken by the Nazi Party in the build up to World War II, the Olympiastadion in recent years has undergone a significant facelift with the addition of a cantilevered roof structure clad in membrane and glass, and has played host to a number of significant sporting events in recent years including the Final of the 2006 World Cup.

2.2.4 Grenoble 1968: The First Temporary Olympic Stadium

The Winter Olympics are generally thought of as the smaller of the two events between the Summer and Winter Games. With fewer nations competing, fewer athletes participating, and generally occurring in more isolated locations (due to the requirement they be near a mountain capable of hosting skiing events), it makes sense that the first temporary Olympic stadium was constructed for a Winter Olympics. Moreover, unlike the Summer Olympics where track and field events are typically held in the event's main stadium, only the opening and closing ceremonies are held in Winter Olympics main stadiums. With these thoughts in mind, in 1968 when the French city Grenoble was selected to host the Winter Games, the decision was made to construct a 60,000 person temporary stadium. Following the closing ceremonies, the entire stadium was deconstructed. Similar processes were used to construct temporary stadiums at both Lake Placid, USA in 1980 and Albertville, France in 1992.

Figure 2. View from the stands during the 1968 Winter Olympics Opening Ceremonies (At Grenoble, Olympics Begin with Grandeur)

2.2.5 Beijing 2008: The Olympics at any Cost

Designed by the Swiss architects Herzog and de Meuron, the Beijing National Stadium (nicknamed "The Bird's Nest") has become a prime example of both the outstanding heights mankind can achieve through modern design and construction practices, as well as the danger short term planning can have on the long term economy of a stadium. Composed of two independent systems, a reinforced concrete seating bowl and the name-sake hollow steel framed "bird's nest" exterior, the Beijing National Stadium was designed to resist the high seismic loads present in the Beijing area. With a capacity of 91,000 and an estimated price tag of roughly $460 million, the Chinese government spared no expense creating a signature structure capable of standing up to any structure around the world on its design merit (Lim, 2012). In the seven years since the 2008 Olympics, the stadium has struggled to find a long term tenant capable of filling the now 80,000 person stadium, and questions still remain as far as how the stadium will find a sustainable way of supporting itself financially moving forward.

2.3 Stadiums and their Current Usage

Since the Sydney Summer Olympics of 2000, an estimated $130 Billion have been spent on staging the Olympics (a price tag encompassing the entire cost of the games including stadium costs, security, advertising, etc.). Of this $130 Billion, roughly $2.8 Billion was spent on the design and construction of the stadium set to host the Opening and Closing ceremonies of the games. Although a large percentage of this estimate was spent in two stadiums (roughly 40% of this estimate comes from Sochi 2014 and Beijing 2008), it is undeniable that host cities for the Olympic Games have spent substantial sums of public money in the planning, infrastructure development, and hosting of these events.

With the size of the upfront investments host nations are making in the sporting infrastructure required to host the Olympics, one logical question to ask might be, what are these stadiums currently being used for? A survey of the 46 different venues who have hosted either the Summer or Winter Opening Ceremonies since 1896 shows that seven have since been demolished or replaced. Of those seven stadiums demolished or replaced, three (Albertville, Lake Placid Equestrian Stadium and Grenoble) were designed as temporary structures intended to be demolished.

Location	Year
London, UK	1908
St. Moritz, Switzerland	1928, 1948
London, UK	1948
Squaw Valley, USA	1960
Grenoble, France*	1968
Lake Placid, USA*	1980
Albertville, France*	1992

* Designed as a temporary stadium

Table 2. Stadiums that have been demolished following use in an Olympics Opening Ceremony

Looking at the remaining 39 previous hosts of the Opening Ceremonies, all but three have found permanent tenants to occupy the venues. In supporting these permanent tenants however, it is important to note that a large majority have required fairly significant reductions in capacity. The London Olympic Stadium, for example, has undergone a $282 Million renovation following the 2012 Games to reduce the capacity of the stadium form its Olympic mode level of 80,000 to its post event mode of 54,000 (Allnutt, 2014). Similarly, in Sydney following the 2000 Summer Games, the ANZ Stadium underwent a $61.4 Million reconfiguration to reduce the stadium's capacity from 110,000 during the Games, to 83,500 after the games (Brookfield Multiplex, 2011).

With the knowledge in mind that most if not all Olympic main stadiums designed as permanent structures will require some sort of scaling down after the event, the primary research goal of this thesis will be to determine whether there is more the field of structural engineering can do to improve the post event adaptability of a stadium through decisions in the design and construction phases.

3 The Stadium

In the field of new construction, especially in the United States, the relationship between the structural engineer and the design architect generally takes on a distinct character based on the type of structure the design team is creating. In the field of bridge design for example, the structure of the bridge is typically thought of as its most important feature, often leading to structural engineering decisions driving the bridge design throughout the delivery of the project. In contrast, in traditional building construction with often more complex programing concerns, the design architect typically is the primary decision maker. In this regard, the field of stadium design has, throughout its history, fallen in an odd middle ground between bridges and buildings. As a complex structural system often involving large cantilevers and long span roof systems, structural engineering decisions can have a major impact on the overall aesthetic and functionality of a stadium. Conversely, with the wide range of programing concerns, as well as the rigorous code requirements that come with assembly faculties housing often thousands of people at a time, architectural decisions can often play a massive role in dictating the structural layout and design of a stadium in a way not often found in other building types. Thus, when looking at stadiums from a structural perspective, it is important to keep in mind the absolute importance of strong collaboration between the architect and the structural engineer throughout the design process. This collaboration becomes even more important when designing Olympic stadiums with the goal of including post event flexibility as a major feature. The stadium must balance aesthetic considerations as the architectural centerpiece of a major sporting event with structural modularity and simplicity to increase the ease with which the stadium can be disassembled or modified when needed.

3.1 Structural Systems

As with the majority of structures inhabited by people, the structural systems that makes up the primary components of a stadium largely fall into three categories. In typical buildings, these systems are the gravity system, lateral system and enclosure. In stadiums, these systems can be broken down into the stadium grandstand or bowl, the roof (if there is one), and the façade. Although from an aesthetic point of view, it is often a major goal to have each of these three systems synergize harmoniously, an emphasis of this thesis will be a focus on minimizing the structural interaction between these systems to reduce the complexity of possible post event renovations.

3.1.1 The Grandstand/Bowl

Supporting the seating and concourse areas most spectators inhabit, the stadium grandstand as a structural system is the system that tends to have the largest impact on the experience spectators have when visiting a stadium. Impacting everything from sightlines to egress methods in the event of an emergency, the overall geometry of a stadium's bowl is rarely controlled by structural concerns, and more often controlled by code or design requirements centered on public safety and the optimal viewing experience.

Modern stadium grandstands are typically designed in either steel or concrete. Although their framing systems vary from stadium to stadium, common structural components that compose most modern stadium grandstands include a floor slab that directly supports seats or bleachers, infill beams that support the slab, and stringer beams raked at an angle determined by either the design architect or the structural engineer. Key considerations for the design of typical stadium grandstands are detailed later in this thesis.

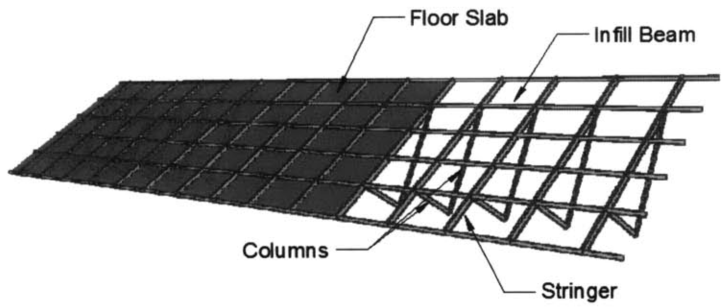

Figure 3. Diagram outlining grandstand terminology

3.1.2 The Roof

Often the most visually striking aspect of many modern stadiums, roofs as structural systems are unique in stadiums due to their typical long span nature. Throughout history, a number of different solutions have been developed to solve the issue of minimizing the visual impact vertical elements (columns) have on spectator's viewing experience. These solutions have included the post and beam structure, the cantilevered structure, the cable net structure and the compression/tension ring structure. Due to the complexity associated with constructing long span roof structures, the choice of the stadium roof structural system can have a major impact on a stadium's overall constructability. Further, this complexity in the construction of a stadium roof system can heavily influence the flexibility of a stadium in its design for post-Olympic usage.

This concept was seen quite distinctly in the issues faced by the London Olympic Stadium owners in their renovation of the stadium following the 2012 Summer Games. In the process of converting the stadium into the future home of the English Premier League's West Ham United, major construction delays and cost increases have arisen due to complications in the deconstruction of the original roof (Allnutt, 2014). With this thought in mind, a balance must be struck between the aesthetic value of a roof and the structural system's complexity if the roof is intended to be modified in the future. Furthermore, the design of the interaction between a stadium's roof and the other structural systems, be it a bearing connection onto the stadium grandstand or a hanging connection from which the stadium façade is suspended, can also play a significant role in improving a stadium's flexibility for post event usage.

3.1.3 The Façade

Along with the roof, a stadium's façade system can play a major role in determining a stadium's primary aesthetic. The Beijing National Stadium designed for the 2008 Summer Olympics earned its namesake "The Bird's Nest" due to its seemingly random steel lattice shape façade that resembles a bird's nest. In stadium façades, designers have been able to integrate perhaps the largest range of materials into stadium design ranging from glass to precast concrete to ethylene tetrafluoroethylene (ETFE). With an eye towards improving structural adaptability for post event usage, designing the stadium's façade as an independently supported structure that does not rely on the grandstand or the roof can help to significantly decrease the complexity any post event renovation may entail.

3.2 Key Design Considerations

While specific design criteria will vary from structure to structure, common design considerations can typically be found across most stadiums, especially in their preliminary design stages. Ranging from structural design loads to more general design criteria with an eye towards a stadium's intended users, these key design considerations are generally a useful starting point from which more detailed analysis can be conducted.

In the United States, stadium designs must meet the specifications of both ASCE-7 or IBC and the ICC 300-2012 *Standard on Bleachers, Folding and Telescopic Seating and Grandstands*. In the context of the design of a flexible structure, specifically the design of a structure intended to be used during an Olympics Opening or Closing Ceremony, the following design considerations take on a slightly different slant than the typical design considerations for most permanent stadium structures. As described later in this thesis, while a stadium structure designed for an Olympics will most likely see its largest live load demands during an Olympics event, the known intended lifespan of 1-2 years may provide the opportunity to take a performance based design approach to the design for wind, snow and seismic loads. Further, looking at the very specific and well known intended use of Olympic

Stadiums during an Opening or Closing Ceremony as it relates to the dynamic response of a structure due to human driven excitation, an opportunity exists to analyze a grandstand's dynamic characteristics in more detail.

3.2.1 Design Loads

The required Dead and Live Loads for stadiums, regardless of their application as a temporary or permanent structure, are outlined in both ASCE-7 and in ICC 300-2012. One important note that should be made about the design live load is the additional requirement in ICC 300-2012 to design for horizontal swaying force applied to each row of seats. The provision reads as follows:

> ***303.4 Horizontal Sway Loads:*** *Bleachers, folding and telescopic seating, and grandstands shall be designed to resist lateral forces produced by the sudden and concerted motion of spectators.*
> ***303.4.1 Sway Parallel to Seating:*** *A horizontal load of 24 pounds per linear foot shall be applied parallel to seating at the footboard level of each row of seating.*
> ***303.4.2 Sway Perpendicular to Seating:*** *A horizontal load of 10 pounds per linear foot shall be applied perpendicular to seating at the footboard level of each row of seating.*

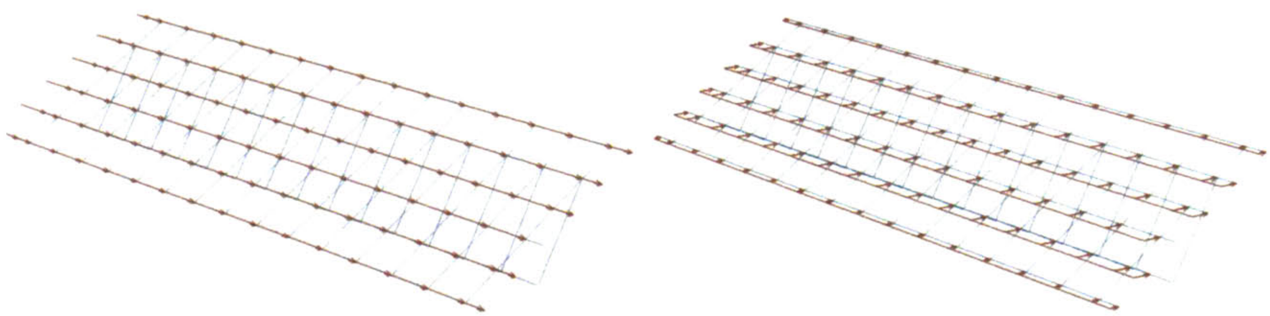

Figure 4. Diagrams illustrating ICC 300-2012 Horizontal Sway Loads

In addition to the load combinations specified in ASCE-7 for LRFD and ASD design the following additional load combinations are required by code for LRFD and ASD design respectively:

> ***303.5.1 Load combinations using strength design or load and resistance factor design.*** *When using strength design or load and resistance factor the following additional load combination must be considered.*

$$1.2D + 1.0L + 1.6Z \quad \text{(Equation 3-1)}$$
$$1.2D + 1.2Rr \quad \text{(Equation 3-2)}$$

> ***303.5.2 Load combinations using allowable stress design.*** *When using allowable stress design the following additional load combination must be considered.*

$$D + 0.75L + 0.75Z \quad \text{(Equation 3-3)}$$
$$D + 0.75Rr \quad \text{(Equation 3-4)}$$

303.5.3 Notations of terms in load combination equations. *The following notations shall, for the purpose of this chapter, have the meanings shown herein.*

D = dead load as defined by the building code
L = live load as defined by Section 303.2
Z = horizontal sway loads as defined by Section 303.4.2 and Section 303.4.3
Rr= guard or handrail loads as defined in Table 303.2

In addition to specifications for dead and live loads, ASCE 7-10 contains the most current design procedures for wind, seismic and snow design. Especially within the design of stadium roofs which are often governed by snow or wind loads, the design procedure outlined in ASCE 7-10 describes the design of structural elements based on a probabilistic approach to environmental forces at clearly defined exceedance probabilities. As will be outlined in the analysis portion of this thesis, this probabilistic approach to determining design snow and wind loads offers the opportunity for a more detailed performance based design of stadium elements within the context of flexible or temporary design (i.e. given a stadium roof or grandstand might be planned to be removed following the Olympic ceremonies, there might be an opportunity to perform a performance based design of the roof or grandstand with less conservative loads, potentially leading to significant material savings).

3.2.2 Dynamics Considerations

Over the past twenty years, considerable research efforts have been invested with the goal of understanding the phenomenon of structural response to human driven dynamics in stadiums. In a paper published as a supplement to the UK's Guide to Safety at Sports Grounds (commonly known as the Green Guide) by The Institution of Structural Engineers in 2001, researchers outline guidelines for vertical natural frequencies "necessary to provide safe and adequate comfort for different categories of use" in various grandstand structural systems. In their study, the authors describe how in addition to using threshold values of fundamental natural frequencies for vibrations as a measure of dynamic performance, factors inducing intended use of the stand and structural damping can also be included to further understand how a stadium grandstand will perform under coordinated crowd induced dynamic loading (The Institute of Structural Engineers, 2001).

As outlined in a paper written by Ginty et al entitled "The frequency ranges of dance-type loads", research evidence would suggest the forcing frequency range for dancing and jumping crowds can be approximated to be within the range of 1.8Hz to 2.3Hz for large groups as occur at pop-concerts

(D. Ginty, 2001). Further, as pointed out by the Institution of Structural Engineers' Working Group, in contrast to sporting events where well synchronized crowed motion is somewhat limited and dynamic concerns are largely focused around the stadiums first fundamental frequency, research evidence suggests events involving musical accompaniment or jumping will tend to excite higher modes as the degree of audience synchronization becomes significantly higher. Within the context of Olympic stadium design, an ability to know more precisely the specific characteristics of the stadium's programing allows stadium designers to more accurately predict the types of dynamic loading a grandstand is likely to experience over its lifetime. For the purposes of this thesis, a 3 Hz frequency threshold will be used as a minimum requirement for grandstand serviceability designs.

3.2.3 Safety and Crowd Egress as Major Drivers of Stadium Geometry

Throughout the 1980s, particularly in soccer stadiums in Europe, a series of tragedies occurred involving spectators being crushed as a result of stampeding crowds. Between the Karaiskakis (1981), Heysel (1985) and Hillsborough (1989) disasters, 156 people died and roughly 1,400 people were injured. In the wake of these disasters, documents like the Talyor Report were enacted to place a larger emphasis on spectator safety at stadiums, recommending the elimination of standing room areas in stadiums (HMSO, 1990). In addition to the *Green Guide*, the governing document in the United Kingdom as it relates to stadium design for spectator safety, the Taylor Report helped reemphasize the building code's priority on human health and safety. Indeed, throughout FIFA's *Football Stadiums: Technical recommendations and requirements* for all stadiums hoping to host FIFA events, the safety of the public is continually emphasised as the primary design requirement of every stadium hosting a FIFA event (FIFA, 2007).

Looking at building codes in the United States as they relate to stadiums, specifically ICC 300-2012, significant guidance is provided as it relates to design requirements with a focus on crowd egress and safety. As pointed out by Allen Gooch in the book *Stadium Engineering*, egress routs in stadiums must be able to accommodate not one, but four different periods of loading with distinct loading patterns: "the pre-match arrival period; half time; post-match egress; and the potential for emergency evacuation that could occur at any time" (Pascoe, 2005). The presence of these four completely different egress periods requires stadium circulation systems to be robust enough to accommodate not only the slower trickle in of spectators entering the stadium before an event, but also the mass rush of spectators leaving at the same time following an event. To design for these requirements, crowd egress has become a major driver in dictating the geometries of concourses and grandstand seating layouts, impacting stadium structural design through requirements for longer spans capable of supporting heavy pedestrian loading.

3.3 Steel as a Structural Material

As mentioned previously, within the realm of stadium grandstand design, two structural materials, steel and concrete have historically dominated the field. Due to their availability, price on a large scale, and the relatively high degree designers understand their mechanical properties, designers choice between the two materials has typically been decided on a per project basis. With an eye towards post event flexibility as is the research goal of this thesis, additional factors beyond the simple economics of the two materials are encouraged to be considered, taking a long term perspective to facility utility beyond the Olympics.

3.3.1 Material Reused Rather than Recycled

In a recent study by the American Iron and Steel Institute, researchers estimated roughly 88% of all steel is recycled, with more than 475 million tons of steel recycled in 2008. This, according to the World Business Council on Sustainable Development, is more than the combined reported totals for several of the world's most commonly used materials including paper, plastic, glass, copper, lead and aluminum (Steel recycling on the rise, 2009). Due to its ability to be melted down and easily reformed into new shapes without any major change in material strength or major shift in material characteristics, steel's ability to be recycled places it at a distinct advantage over concrete from a sustainability perspective.

To further this advantage, designers have begun seeing opportunities to reuse structural steel elements rather than recycle them (two examples included the London Olympic Stadium reusing roughly 4,000 tonnes of discarded gas pipeline and the modular construction of structures using steel shipping containers) (Buro Happold, 2015). Beyond reducing the industry's dependence on mining virgin materials, the reuse of steel elements eliminates the energy cost of recycling (processing, melting down and reforming steel elements). This ability to reuse structural steel elements illustrates the opportunity stadium designers have to design stadiums that are both sustainable from an economic and environmental perspective.

3.3.2 Detachable Connection Design

In keeping with the idea of improving stadium sustainability through the reuse of structural steel elements, structural engineers play a unique role in increasing the ease with which elements can be reused through connection design and detailing. Although the most common practices today within steel connection design typically involve the use of bolted or welded connections, a possible area for future investigation beyond the scope of this thesis might be the design and testing of steel connections involving non-destructive clamps that minimize the use of fixings to structural steel elements, fixings that potentially limit that steel elements future uses. Within the realm of conventional construction practices however, through the use of bolted rather than welded connections, steel elements can more easily be disassembled in the post event renovation process. This specification of bolted rather than welded connections however comes at the cost of reduced structural stiffness through the design of pined rather than fixed connections. The structural implications of bolted versus welded connections will be analyzed in detail in the next chapter of this thesis.

4 The Structural Implications of Element End Conditions in Stadium Grandstand Design

Within steel connection design, the trade-off between structural stiffness and constructability surrounding bolted versus welded connections has been well documented. In a paper written by Farkas et al. at the University of Miskolc in Hungry, researchers analyzed the economic implications involved with bolted and welded connections in various configurations in braced frame structures. In their findings, Farkas et al. determined, for a given level of strength and serviceability performance, bolt connected structures are roughly 7% more expensive in terms of material than weld connected structures. When factoring in the cost of constructability (i.e. the cost of labor) however, bolt connected structures are between 6 and 17% cheaper than weld connected structures in total cost (materials plus labor cost) (J. Farkas, 2003).

This chapter will outline a study conducted on two different stadium geometries, an elliptical and rectangular grandstand, analyzing the effect of connection type on structural design in stadium grandstands. The first portion will describe the procedure for the strength design of a representative 15 bay stadium grandstand segment with either an elliptical or rectangular (straight) shape overall in plan, and the second portion will describe the procedure for stiffening the strength design on a full stadium to reach a fundamental frequency of at least 3 Hz.

4.1 Methodology

4.1.1 Case Study Geometries

In setting the overall geometry of a stadium grandstand, a number of factors outside the control of the structural engineer come into play. As Culley and Pascoe describe in their book *Stadium Engineering*, the decision to determine the grandstand rake, or the inclination angle of the stringer takes the form of a balance between improved sightlines and the boundaries of human comfort. Describing typical upper bounds for rake limits, beyond which humans begin to feel a sense of vertigo, the United Kingdom's Green Guide outlines 34° as a general limit for most large scale stadiums (Department for Culture, Media and Sport, 2008). Thus for the purposes of this case study both the elliptical and rectangular stadium sections considered use 34° as the rake for the stadium's stringer elements.

In addition to determining stringer rake based on Green Guide recommendations, stringer spacing and length were roughly determined based on Green Guide requirements for minimum seat width and depth, (estimating 0.5m (1.64 ft) per seat width and 0.85m (2.79 ft) seat depth). The grandstand cross section was determined such that 30 rows of seats could be supported along each stringer length. Stringer support locations were determined with the goal of minimizing moment and deflection in the beam. By selecting stringer support locations such that the maximum positive moment at the supports equaled the maximum negative moment at the stringer mid-span, the stringer is supported at $0.5(\sqrt{2}-1)L$ from either end. A detailed spreadsheet outlining node locations for each of the case studies analyzed can be found in Appendix A.

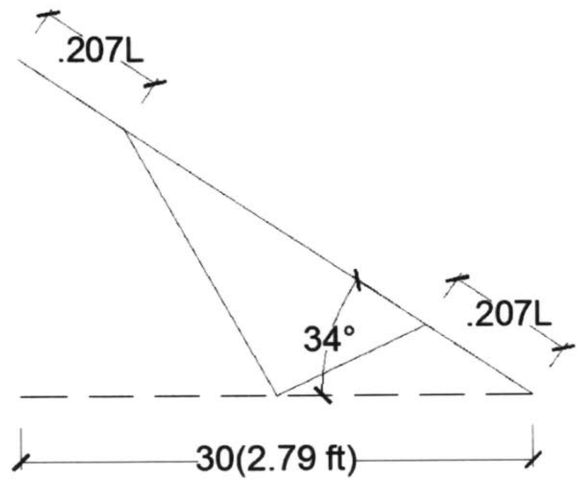

Figure 5. Case study cross section in elevation

The overall geometry of the elliptical case study is roughly based on the geometry used in the London Olympic Stadium (an ellipse roughly 1050 ft long and 886 ft wide). The rectangular case study follows the same dimensions (see Figure 6).

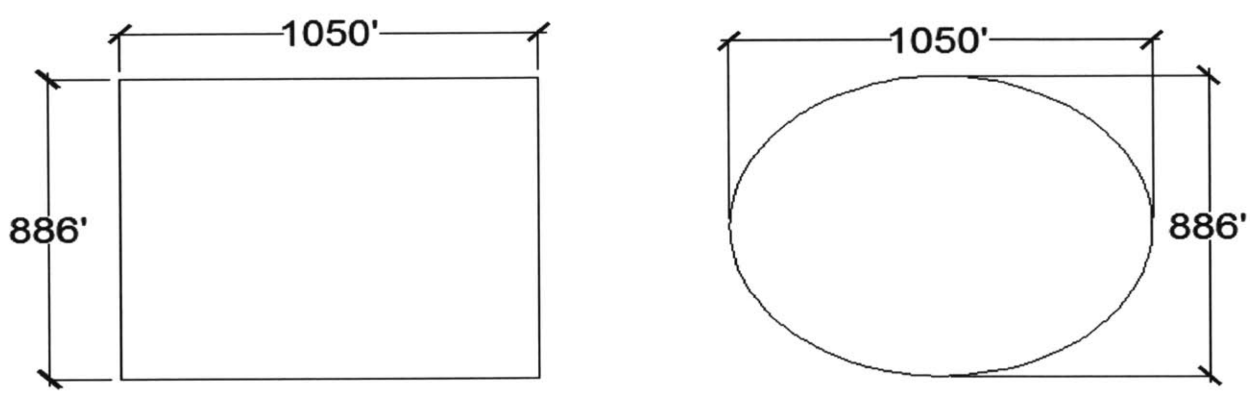

Figure 6. Plan view of elliptical and rectangular case studies with dimensions

4.1.2 Strength Design Support Conditions

To simplify the model necessary to analyze the stadium grandstand, for the strength design of each stadium layout, a representative 15 bay sections was modeled and designed using the code loads outlined in Chapter 3. For the elliptical layout, to study the effect global grandstand curvature can have on structural stiffness, two separate 15 bay segments were analyzed; one taken as a portion of the ellipse with the largest curvature, and the other taken as a portion of the ellipse with the smallest curvature. The modeled grandstands included rigid supports at the base of each of the V-shaped vertical supports (see Figure 7 below). To capture the effect of adjacent stadium segments, each of the end beams were cut in half and restrained in their local x and y directions.

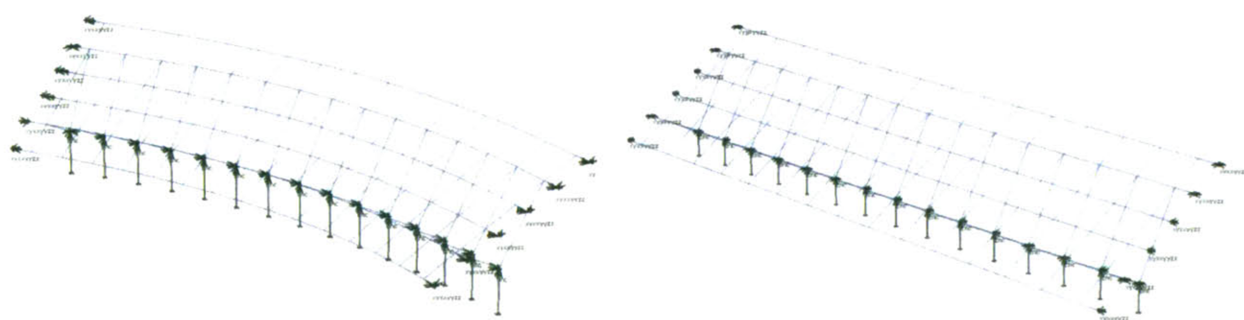

Figure 7. Grandstand support conditions

4.1.3 Element End Conditions for Bolted and Welded Connections

In order to analyze the effect of bolted or welded connections on structural design in grandstands, the primary variable permeated across the various studies was the end condition of each of the infill beams within each case study section. By releasing the infill beams of moment at each of the beam to stringer connections (see Figure 8 for an illustration), a bolted connection capable of transferring shear only was modeled. Although in reality bolted connections do provide some degree of rotation restraint, for the purposes of this thesis, they are not relied upon to contribute adequate moment resistance. Welded connections on the other hand are modeled as fixed connections capable of complete moment transfer across a connection.

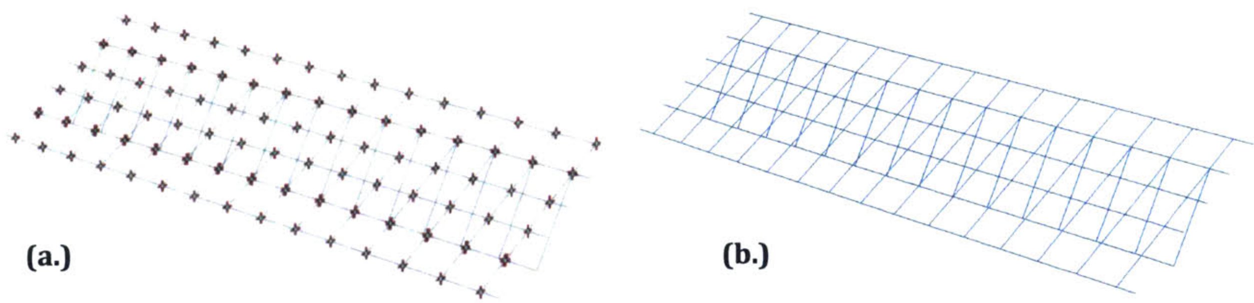

Figure 8. Illustration of element moment releases in the local yy and zz directions for (a.) bolt connected and (b.) weld connected case studies

4.1.4 Strength Design Procedure

Once each of the three case study sections were modeled applying loads outlined in Section 3.2.1 according to tributary areas, a linear static analysis was run to determine element forces and support reactions. Models were validated using a combination of simplified hand calculations and a visual inspection of the analysis deflected shapes, and moment and axial diagrams. Using element force and moments obtained from the finite element models, each of the element sections were designed using standard AISC wide flange sections. Strength based element designs for each of the three different case study sections can be found in Appendix B.

4.1.5 Serviceability Design Procedure

In order to analyze the effect the connection type has on a grandstand's ability to meet the serviceability requirements outlined in Chapter 3, a full finite element model of an elliptical grandstand was constructed with the geometry described in Section 4.1.1. The full model was analyzed in order to capture global mode shapes that were more likely to control dynamic design rather than local mode shapes. Element sizes were selected to match the strength design sections determined in Section 4.1.4, and a modal analysis was conducted on the finite element model after including an 8" concrete slab (150 pcf concrete) for additional mass. Given that the grandstand did not meet the 3 Hz minimum design threshold, what followed was the development of a strategy to increase the overall stiffness of the grandstand while minimizing the additional weight added to the structure (frequency increases with stiffness and decreases with mass).

4.1.6 Serviceability Support Conditions

As the goal of the serviceability analysis was to determine the effect element connection types have on global modal properties, specifically on the grandstand modal properties within the plane of the seating bowl, the V-shaped vertical supports modeled in the strength based design were replaced with pin supports in the finite element model. These restraints allowed rotation but not translation about the vertical supports. See Figure 9 for an illustration of the model support condition applied to both bolted and welded models. The node locations for the elliptical grandstand match the node locations outlined in Appendix A rotated 360°.

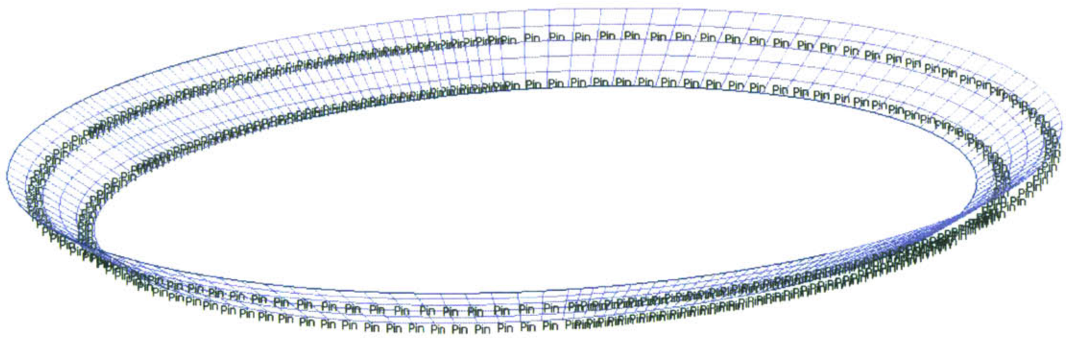

Figure 9. Grandstand support conditions for serviceability analysis

4.1.7 Grandstand Stiffening Strategy

Iteratively increasing element stiffness by factoring elements' strong axis moment of inertia, it was determined early on that stringer stiffness rather than infill beam stiffness had a significantly larger impact on increasing the overall global stiffness of the structure. By treating stringer stiffness as a variable to be optimized, the goal of the stringer stiffening strategy became to minimize the amount of artificial stiffness added to the structure (i.e. stiffness added to the structure without increasing weight or any element dimensions) while at the same time reaching a minimum performance criteria of 3 Hz fundamental frequency. Given the results determined in Section 4.1.4 where grandstand areas incorporating larger curvature tend to be much stiffer for equivalent member cross sections, the general stringer stiffening strategy was focused on distributing stiffness such that the first mode was activated above 3 Hz and occurred along the long edge of the ellipse. Stringers were organized into three groups (seen in Figure 10) and a weighted score system was developed as follows:

$$Wt'd\ Score = \sum \gamma_i \times \#\ Elements\ in\ Group\ i \qquad \text{(Equation 4-1)}$$

$$\gamma_i = factor\ on\ I_{yy}$$

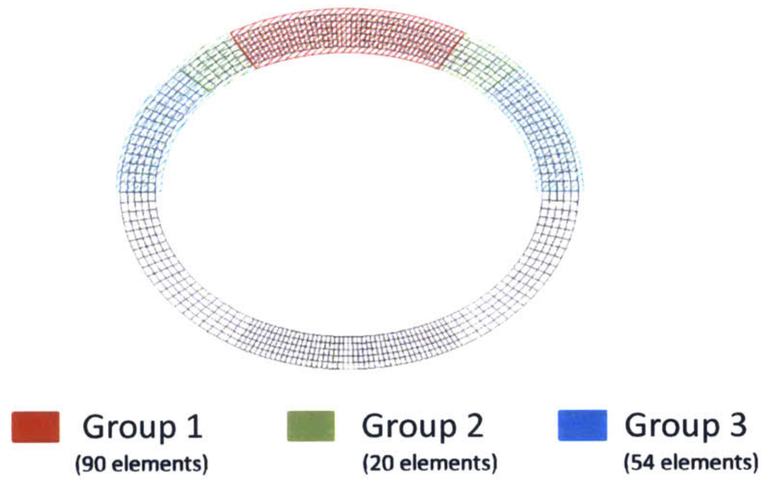

Figure 10. Layout of stringer groups

4.2 Results

4.2.1 Strength Design Results

Using the procedure outlined in Section 4.1.4, the three different 15 bay stadium grandstands were modeled and designed using both fixed and moment released end fixities on the infill beams. The results of the study have been summarized in Table 3:

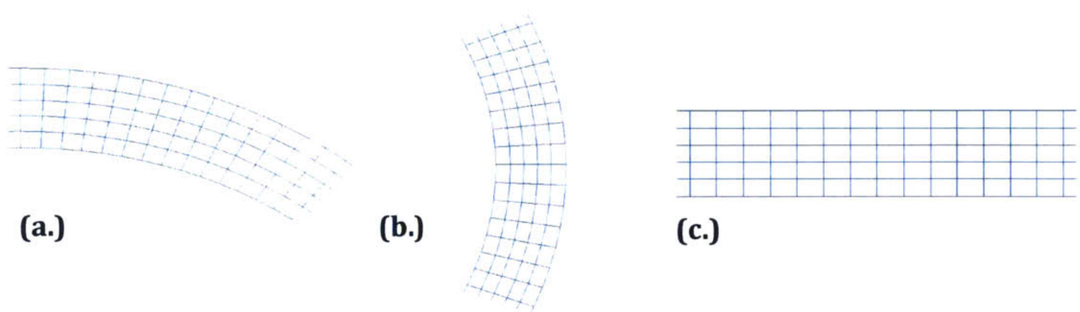

Figure 11. (a.) Ellipse 1 (b.) Ellipse 2 (c.) Straight

Case Study	Connection Type	Steel Wt per Seat (lbs/seat)	Steel Wt per Sq Ft (lb/ft^2)	Difference Between Welded and Bolted (lb/ft^2)	Percentage Difference
Ellipse 1	Welded	48.1	8.94	-	-
Ellipse 1	Bolted	50.2	9.33	0.40	4.27%
Ellipse 2	Welded	48.7	8.96	-	-
Ellipse 2	Bolted	51.2	9.41	0.45	4.81%
Straight	Welded	47.8	8.96	-	-
Straight	Bolted	48.9	9.16	0.20	2.19%

Table 3. Grandstand strength design summary

Looking at the result of the three different case studies and the material quantities associated with each of the six different designs, the first thing that jump outs is the fact that bolt connected case studies categorically required larger steel elements to support the same loads than weld connected case studies. This matches intuition and previous works mentioned earlier in this thesis (Farakas et al.) as welded (fixed) end conditions increase stiffness, reducing deflections and element moment and axial demands. However, looking specifically at the elliptical sections, the result of roughly 4.5% additional material required for bolt connected elliptical grandstands combined with Farakas et al.'s findings on the cost trade-off between bolted and welded connections would seem to indicate the benefits of bolted connections significantly outweigh the costs. Further, as referenced in Section 3.3.2, with an eye towards improving post event flexibility of a stadium, this 4.5% increase in material becomes even less significant in light of the additional utility bolted connections provide.

4.2.2 Serviceability Design Results

Using the methodology outlined in Section 4.1.5, the results of the grandstand stiffening study are as follows for the bolt connected case study:

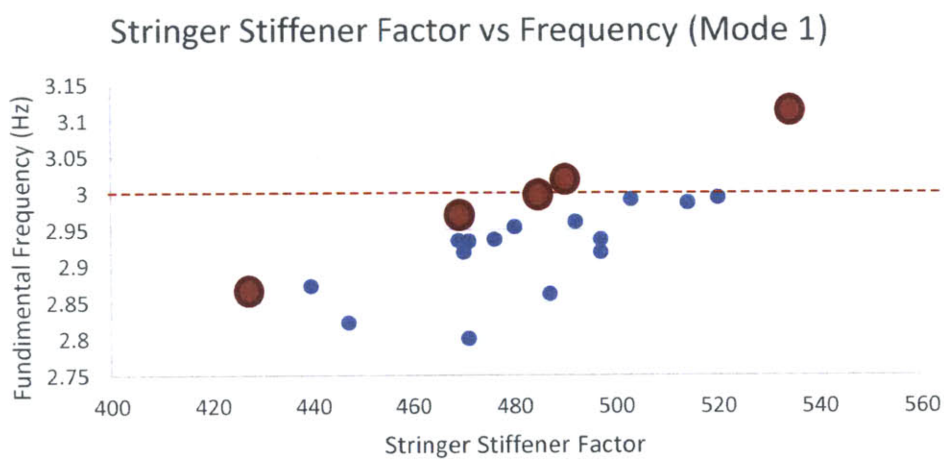

Figure 12. Plot of stringer stiffening factor vs the first fundamental frequency

While, from inspection, a linear fit is perhaps not the best indication of the data distribution, the stringer stiffener factor is clearly at least positively correlated with fundamental frequency. Selecting five points (distributions of stiffness) that appear to outperform other stiffness distributions (highlighted in red), the effect of beam end fixity (pinned versus fixed) was evaluated for these five distributions of stiffness. The results are as follows:

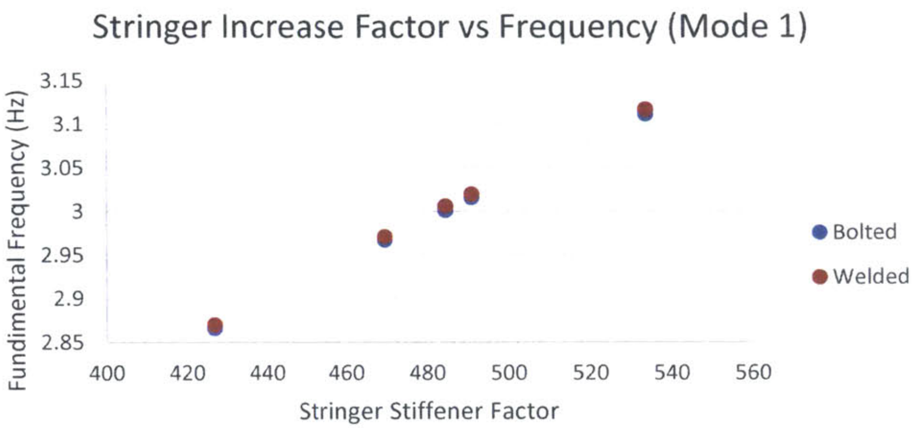

Figure 13. Selected bolted and welded grandstand fundamental frequencies

Comparing the results at these five points between the bolted and welded case studies, it would appear, while the weld connected case study consistently performed better for equivalent stiffness factors (for the same reason the welded gravity designs required less material), this difference is not substantial. The differences in frequencies between the bolted and welded grandstands are summarized in Table 4:

Stiffener Factor	Mode 1 (Hz)	Mode 2 (Hz)	Mode 3 (Hz)
427	0.004	0.004	0.006
469.7	0.004	0.003	0.005
484.645	0.005	0.004	0.005
491.05	0.004	0.003	0.004
533.75	0.006	0.005	0.004
Average:	0.0046	0.0038	0.0048

Table 4. Difference between bolted and welded grandstand natural frequencies for equivalent stiffener factors

5 Performance Based Design of an Elliptical Roof Truss System

With the growth of Load Resistance Factor Design (LRFD), building codes have begun to place a larger emphasis on a probabilistic approach to building design. Many of the design loads in the current building codes including snow, wind and seismic loads specified in both ASCE 7-10 and IBC 2012 are based on historic records with data used to estimate time related exceedance probabilities.

Within the context of Olympic stadiums, when designing structural systems for stadiums with a predetermined deconstruction date (i.e. designing a stadium roof with the intention of removing the roof following the Games), the time related nature of the current building codes offers an opportunity for a more precise performance based design approach to systems most often controlled by wind and snow loads. Indeed, by accepting the fact that code design loads are intended for use in the design of permanent building systems with life spans on the order of ten to twenty times longer than an Olympics stadium takes to construct, a performance based design procedure has the potential to provide significant design savings while at the same time more accurately predict the loads a temporary structure is likely to see over its lifetime.

This section of the analysis procedure will describe the performance based design of an elliptical roof truss system for a stadium roof in Boston, MA designed for wind and snow loads. The general procedure will be to design two different roofs; one to current building codes (ASCE 7-10), and one to a 2% failure probability over 3 years. The specifics of this procedure for use in wind and snow design will be outlined in the following subsections.

5.1 Methodology

5.1.1 Design Wind Loads

In the literature describing the methodology behind the development of the wind speed maps currently used to determine wind loads in ASCE 7-10, code authors Cook et al describe the process by which the various Occupancy Categories wind speeds were assigned (Cook & al., 2011). Using the historic 50 year return period wind event (the nominal design wind speed) as a baseline value, these wind speeds are factored according to an equation defined in ASCE 7-95 as follows to achieve the various Occupancy Category wind speeds:

$$V_T/V_{50} = [0.36 + 0.1\ ln(12T)] = \sqrt{W_{LF}} \qquad \text{(Equation 5-1)}$$

$$T = 0.00228\ exp(10\sqrt{W_{LF}}) \qquad \text{(Equation 5-2)}$$

$$W_T = C_F V_T^2 = C_F V_{50}^2 W_{LF} \qquad \text{(Equation 5-3)}$$

V_{50} = 50 year return period wind speed (nominal design wind speed)
V_T = T year return period wind speed
T= expected return period (1/T = exceedance probability)
W_{LF} = wind load factor
W_T = limit state wind load
C_F = component/structure specific coefficient that includes the effects of building height, geometry, terrain, etc.

Using W_{LF} = 1.6 as specified in ASCE 7-05 to determine Occupancy Category II wind speeds (speeds corresponding to a return period of about 709 years), the 50 year return period wind speed can be solved for using the ASCE 7-10 wind speed maps for any locations in the United State (map attached in Appendix C). From there, Equation 5-2 can be used to determine the W_{LF} for any desired return period T, and Equation 5-1 can be used to calculate V_T for that desired return period.

Applying these equations to determine V_{50} for Boston, MA, from the ASCE 7-10 map for Occupancy Category II:

$$V_{50} = \frac{V_{Cat\,II}}{\sqrt{1.6}} = \frac{135}{\sqrt{1.6}} = 106.7\ mph$$

Modeling the wind exceedance distribution as a binomial distribution, the minimum return period required to satisfy the design criteria of 2% exceedance over 3 years for wind can be calculated as follows:

$$\Pr(X \geq 1) = 1 - \Pr(X = 0) \rightarrow \Pr(X \geq 1) = 1 - \left(1 - \tfrac{1}{T}\right)^n \qquad \text{(Equation 5-4)}$$

$$\rightarrow T = [1 - (1 - \Pr(X = 0))^{\frac{1}{n}}]^{-1} = [1 - (1 - (0.02))^{\frac{1}{3}}]^{-1} = 149\ years$$

Combining these two results:

$$\sqrt{W_{LF}} = [0.36 + 0.1\ln(12 * (149))] = 1.109$$

$$\rightarrow V_{149} = V_{50}\sqrt{W_{LF}} = (106.7)(1.109) = 118.3\ mph$$

In an attempt to design the stadium roof conservatively with minimal knowledge about the physical site and the stadium design, the following constants are assumed to determine wind load W_T:

K_d = 0.85

K_{zt} = 1.0

K_z = 1.28 (estimate height above ground level, z = 250, Exposure Class B)

G = 0.85

$C_p = 0.9$ (h/L=0.123<0.5, Θ<10°)

$GC_{pi} = -0.55$ (Roof opening makes the building Partially Enclosed)

Using the procedure outlined in Chapter 27 of ASCE 7-10 and synthesizing these results with the equations described above:

$$q = 0.00256 K_d K_z K_{zt} V^2 \qquad \text{(Equation 5-5)}$$

$$W_T = q(GC_p - GC_{pi}) = C_F {V_T}^2 \qquad \text{(Equation 5-6)}$$

$$\rightarrow C_F = 0.00256 K_d K_z K_{zt}(GC_p - GC_{pi}) = 0.00366$$

$$W_{149} = 0.00366(118.3)^2 = 51.3 \text{ psf}$$

It should be noted the primary goal of this analysis is to compare relative design loads between code and performance based design values (i.e. the important result is the relative difference in loads, not the actual loads themselves).

As determined by ASCE 7-10, stadium structures classify as Occupancy Category III. Thus the code design load for the roof structure is:

$$W_{\text{Cat III}} = 0.00366(145)^2 = 77.0 \text{ psf}$$

5.1.2 Design Snow Loads

In contrast to wind loads where all of the time related probability characteristics of the design loads are explicitly detailed, in codes outlining the process to calculate snow loads, the process by which uncertainty is built into design loads is much less transparent. The current ASCE 7-10 design snow load procedure involves the use of a map which details the ground snow pressure at various locations across the United States, reporting a ground snow load with a 2% exceedance probability annually based on historic snowfall data. This snow load is then increased by a factor of 1.6, a number selected by code officials with no clear relationship to any exceedance probability. In order to conduct a performance based analysis of a stadium roof structure designed to a 2% probability of failure over 3 years, the distribution used to generate code snow loads based off historical weather data had to be recreated.

Using data provided by the National Ocean and Atmospheric Administration (NOAA) for the weather station located at Boston's Logan Airport (the "first order" weather station used to determine ASCE 7-10 code snow loads), the maximum annual water equivalent of snow depth (WESD) was determined for each year from 1951 to 1992 (timespan outlined in the commentary to ASCE 7-10's snow load chapter) (National Oceanic and Atmospheric Administration, 2015). The raw data can be found in Appendix D. The WESD is a standard proxy for snow depth that eliminates the requirement for knowledge of snow density which is not always recorded.

In an attempt to determine the proper distribution to fit the historic data to, two distributions were considered, the log normal distribution outlined by ASCE 7-10 in the snow load commentary and the Fisher-Tippet Type I distribution outlined in the Ellingwood and Redfield paper "Probability Models for Annual Extreme Water-Equivalent Ground Snow" (Ellingwood & Redfield, 1984). The parameters of the two models fit to the historic WESD data, x are as follows:

$$\hat{\mu} = 2.28\ in;\ \hat{\sigma} = 2.34$$

$$\rightarrow \hat{\lambda} = E[\ln x] = 0.592;\ \hat{\xi}^2 = Var[ln\ x] = 0.616$$

For log normal:

$$F_{LN}(x) = \Phi\left[\frac{lnx - \lambda}{\xi}\right] \quad \text{(Equation 5-7)}$$

Manipulating Equation 5-7 to estimate the WESD with an N year MRI (mean return interval) value:

$$\hat{x}_N = \exp(\hat{\lambda} - \hat{\xi}\Phi^{-1}\left(1 - \frac{1}{N}\right)) \quad \text{(Equation 5-8)}$$

For Type I:

$$\alpha = \frac{1}{\sqrt{\sigma^2 * \frac{6}{\pi^2}}} \rightarrow \hat{\alpha} = \frac{1}{\sqrt{(2.34)^2 * \frac{6}{\pi^2}}} = 0.549$$

$$u = \mu - \frac{0.5772}{\alpha} \rightarrow \hat{u} = (2.28) - \frac{0.5772}{(2.34)} = 1.22$$

$$F_I(x) = \exp(-\exp(-\alpha(x - u))) \quad \text{(Equation 5-9)}$$

Manipulating Equation 5-9 to estimate the WESD with an N year MRI (mean return interval) value:

$$\hat{x}_N = \hat{u} - \frac{1}{\hat{\alpha}}\ln(-\ln\left(1 - \frac{1}{N}\right)) \quad \text{(Equation 5-10)}$$

As mentioned previously, ASCE 7-10 specifies ground snow loads corresponding to a 2% annual exceedance probability (MRI = 50 year). Fitting the WESD data to both the log normal and Type I distributions yields the following result:

Log normal distribution:

$$\hat{x}_{50} = \exp(0.592 - 0.616\Phi^{-1}\left(1 - \frac{1}{(50)}\right)) = 6.40\, in$$

$$S_{50} = 6.40\gamma_w = 6.40(62.4\frac{lb}{ft^3}) = 33.3\, psf$$

Type I distribution:

$$\hat{x}_{50} = (1.22) - \frac{1}{(0.549)}\ln(-\ln(1 - \frac{1}{(50)})) = 8.33\, in$$

$$S_{50} = 8.33\gamma_w = 8.33(62.4\frac{lb}{ft^3}) = 43.3\, psf \approx 45psf$$

The log normal distribution load value appears to match the value found in Table C7-1 of the ASCE 7-10 design code, while the Type I value appears to match the nominal design load for Boston, MA currently found in the Massachusetts building code. Thus, the Type I distribution tuned to the historic WESD data was selected as the distribution to calculate snow loads for the roof performance based design analysis. Using the same performance criteria outlined for wind (2% chance of exceedance over 3 years modeled as a binomial distribution):

$$\Pr(X \geq 1) = 1 - \Pr(X = 0) \rightarrow \Pr(X \geq 1) = 1 - \left(1 - \frac{1}{T}\right)^n$$

$$\rightarrow T = [1 - (1 - \Pr(X \geq 1))^{\frac{1}{n}}]^{-1} = [1 - (1 - (0.02))^{\frac{1}{3}}]^{-1} = 149\, years$$

$$\hat{x}_{149} = (1.22) - \frac{1}{(0.549)}\ln(-\ln(1 - \frac{1}{(149)})) = 10.33\, in$$

$$S_{149} = 10.33\gamma_w = 10.33\left(62.4\frac{lb}{ft^3}\right) = 53.7psf$$

Further, by setting $F_I(x) = 1-1/N$ equal to Equation 5-9, the return period associated with 1.6 times the code design load can be solved for as follows:

$$F_I(x) = \exp(-\exp(-\hat{\alpha}(1.6\hat{x}_{50} - \hat{u}))) = 1 - \frac{1}{N} \rightarrow N = 770\, years$$

5.1.3 Roof Design Procedure

Using the same elliptical envelope in plan as modeled in the grandstand case study evaluation, an entire elliptical truss ring roof system was modeled in a finite element software using the following cross section (spreadsheet detailing roof node locations can be found in Appendix E):

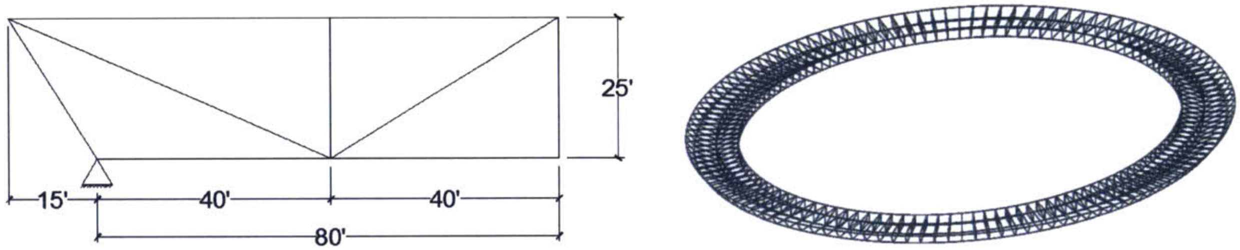

Figure 14. (a) Roof cross section geometry and support conditions (b) Complete model image

The roof was pin supported at each of the nodes at the base of each cross section along the outer most elliptical ring (see Figure 14a. above). All elements were modeled as beam elements capable of carrying bending, and the cross section for every element in the model was designed as identical solid A992 grade steel rectangles. Given that the goal of this study was to determine the relative effect of using performance rather than codes based loads on structural material quantities, the roof structure was designed coarsely, sizing elements for Euler buckling, tension failure, strong and weak axis bending and deflection (L/100 criteria). The loads used are summarized in Table 5 below:

	Performance:	Code:
Tributary width (ft)	25.3	25.3
Membrane SW (klf)	0.00284	0.00284
Snow (klf)	1.26	1.05
Wind (klf)	1.30	1.95
Roof Live (klf)	1.62	1.62

Table 5. Performance and code based design loads

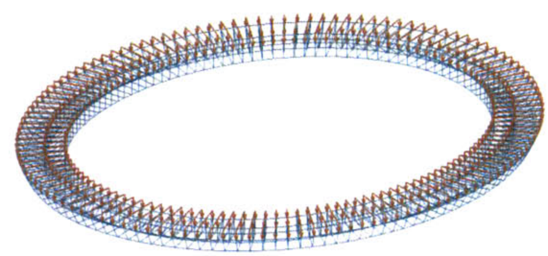

Figure 15. Applied roof loads

A tributary area approach was used based on a constant tributary width of 25.3 ft. For dead load, a 0.0099 inch thick coated PTFE membrane was assumed. Design loads were applied as line loads to the radiating beam elements on the upper surface of the roof as illustrated in Figure 15. Once the following LRFD design equations were assigned for the strength design of the roof, a linear static analysis was run to determine element forces and moments.

[LRFD design equations] → [Performance based design equations]

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W) \rightarrow 1.2D + 1.0S + 0.5W$

$1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R) \rightarrow 1.2D + 1.0W + 0.5S$

$0.9D + 1.0W$

From these element forces and moments, members were sized based on the critical compression, flexural and tension elements. Finally, total material quantities were calculated for both the code and performance based designs and the results were compared.

5.2 Results

Using the procedure outlined in Section 5.1.3, the following controlling code and performance load cases' max tensile, compressive, and bending forces were determined from the finite element model:

	Performance:	Code:
Pr (kips):	-2220	-3056
Fr (kips):	346	478
Mzz (weak axis) (kip-ft):	612	820
Myy (strong axis) (kip-ft):	126	184

Table 7. Roof case study element demand summary

Sizing the roof section for strength, however, the minimum weight cross section does not meet the deflection criteria of L/100 for either the performance or code case studies. Thus, sizing the roof section for deflection yields the following minimum weight values:

	Performance:	Code:
Strength Section:	7 x 12	8 x 13
Deflection Section:	29 x 6	32 x 7
Max Deflection:	L/102	L/100
Roof Steel Weight (kips):	36,550	47,060
Percent Difference:	-	22%

Table 6. Roof case study designed section summary

By designing for performance rather than code based loads, a 22% reduction in material weight was achieved. Although the performance criteria used in this analysis was somewhat arbitrary, the results of this study indicate the benefits a performance based approach can have in the design of structures with the intended life-span on the order of an Olympics cycle, providing designers with the ability to explicitly specify an acceptable level of risk.

6 Conclusions

In January 2015, the city of Boston, MA won the United States Olympic Committee bid to host the 2024 Summer Olympics. Among other things, a key component of the Boston 2024 proposal is an emphasis on "modular construction" (what this thesis has called flexible construction) in the design of the Olympic stadium. In their proposal, the organizers of Boston 2024 state their goal of hosting an event that is not only well executed, but also leaves a positive and sustainable legacy beyond summer 2024 (Boston 2024, 2014). In an article critiquing one proposal for a completely temporary stadium for the Boston Summer Olympics, architect and stadium designer Marc Schulitz stated the following: "No one should think it's cheaper than building a [permanent] stadium; the requirements are the same for life safety, fireproofing, egress — everything has to work and be to code, meaning the way you build it is not going to be that much different from a permanent stadium" (Levenson, 2015).

Given the findings of this thesis, it would appear Mr. Schulitz's statement may not be entirely accurate. Through an analysis of the impact beam end fixities have on structural design, this thesis found only modest increases in material costs for bolted connections in comparison to welded connections in both strength and serviceability design. Through a process of performance based design, taking a more precise, probabilistic approach to the way we specify design loads, significant material savings were found through the design of a case study roof.

Massive sums of public funding are currently being spent on stadiums around the world to host events like the World Cup and the Olympics. From Rio, to Pyeongchang, to Tokyo, in preparation for the next three Olympic Games, cities are investing heavily in sporting infrastructure with the hope that these investments will provide economic benefits to their communities in the long run. With this fact in mind, it is the responsibility of all parties involved in the delivery of those projects to produce work that not only adequately meets requirements for life safety, but also provides cities with facilities that are sustainable beyond the Closing Ceremonies. As structural engineers, Olympic stadium design presents an exciting opportunity to develop new and innovative ways to improve a stadium's capability for post-event renovation.

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Appendix A

Ellipse 1 Geometry:

Beam 1		
x (ft)	y (ft)	z (ft)
-12.2	442.9	54.5
0.0	0.0	54.5
24.5	442.5	54.5
49.0	441.1	54.5
73.3	438.7	54.5
97.6	435.3	54.5
121.7	430.9	54.5
145.6	425.6	54.5
169.3	419.3	54.5
192.7	412.1	54.5
215.8	403.9	54.5
238.5	394.7	54.5
260.8	384.5	54.5
282.6	373.3	54.5
303.9	361.2	54.5
324.6	348.2	54.5
334.7	341.3	54.5

Beam 2		
x (ft)	y (ft)	z (ft)
-11.8	426.1	43.3
0.0	426.3	43.3
23.6	425.8	43.3
47.1	424.4	43.3
70.6	422.1	43.3
93.9	418.9	43.3
117.2	414.8	43.3
140.2	409.8	43.3
163.0	403.8	43.3
185.6	396.9	43.3
207.9	389.1	43.3
229.8	380.3	43.3
251.4	370.6	43.3
272.5	360.0	43.3
293.1	348.4	43.3
313.2	335.9	43.3
323.0	329.3	43.3

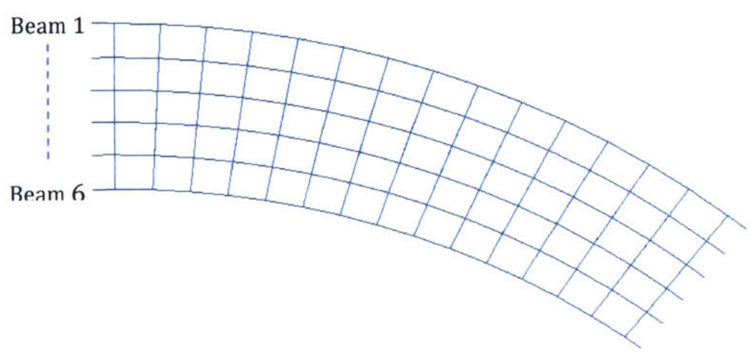
Beam 3		
x (ft)	y (ft)	z (ft)
-12.2	410.4	32.6
0.0	410.5	32.6
22.7	410.0	32.6
45.4	408.7	32.6
68.0	406.6	32.6
90.5	403.5	32.6
112.9	399.6	32.6
135.1	394.8	32.6
157.1	389.2	32.6
178.9	382.6	32.6
200.5	375.1	32.6
221.7	366.8	32.6
242.5	357.6	32.6
263.0	347.4	32.6
283.0	336.3	32.6
302.4	324.4	32.6
312.0	318.0	32.6

Beam 4		
x (ft)	y (ft)	z (ft)
-10.9	394.6	21.9
0.0	394.7	21.9
21.8	394.3	21.9
43.6	393.0	21.9
65.4	391.0	21.9
87.0	388.1	21.9
108.6	384.4	21.9
130.0	379.9	21.9
151.2	374.5	21.9
172.2	368.3	21.9
193.0	361.2	21.9
213.5	353.3	21.9
233.7	344.5	21.9
253.4	334.8	21.9
272.8	324.2	21.9
291.7	312.8	21.9
300.9	306.8	21.9

Beam 5		
x (ft)	y (ft)	z (ft)
-10.5	378.8	11.3
0.0	378.9	11.3
21.0	378.5	11.3
41.9	377.3	11.3
62.8	375.4	11.3
83.6	372.7	11.3
104.3	369.2	11.3
124.9	364.9	11.3
145.3	359.9	11.3
165.5	354.0	11.3
185.6	347.3	11.3
205.3	339.8	11.3
224.8	331.4	11.3
243.9	322.2	11.3
262.6	312.2	11.3
280.9	301.3	11.3
289.8	295.5	11.3

Beam 6		
x (ft)	y (ft)	z (ft)
-10.0	362.0	0
0	362.13	0
20.0	361.8	0
40.0	360.7	0
60.0	358.9	0
79.9	356.4	0
99.7	353.1	0
119.4	349.1	0
139.0	344.3	0
158.4	338.8	0
177.7	332.5	0
196.7	325.4	0
215.4	317.6	0
233.8	308.9	0
251.8	299.3	0
269.5	289.0	0
278.1	283.5	0

Column Base		
x (ft)	y (ft)	z (ft)
-	-	-
0.0	402.6	0
22.3	402.1	0
44.5	400.9	0
66.7	398.8	0
88.8	395.8	0
110.7	392.0	0
132.5	387.4	0
154.2	381.8	0
175.6	375.4	0
196.7	368.2	0
217.6	360.0	0
238.1	351.0	0
258.2	341.1	0
277.9	330.3	0
297.0	318.6	0
-	-	-



Ellipse 2 Geometry:

Beam 1		
x (ft)	y (ft)	z (ft)
480.5	178.6	54.5
486.0	167.6	54.5
496.0	145.3	54.5
504.6	122.3	54.5
511.8	98.9	54.5
517.4	75.0	54.5
521.5	50.9	54.5
524.1	26.5	54.5
525.0	2.0	54.5
524.3	-22.4	54.5
522.1	-46.8	54.5
518.2	-71.0	54.5
512.8	-94.9	54.5
505.9	-118.4	54.5
497.5	-141.5	54.5
487.7	-163.9	54.5
482.3	-174.9	54.5

Beam 2		
x (ft)	y (ft)	z (ft)
464.8	172.7	43.3
470.1	162.2	43.3
479.9	140.5	43.3
488.3	118.4	43.3
495.3	95.7	43.3
500.8	72.6	43.3
504.9	49.3	43.3
507.3	25.7	43.3
508.2	2.0	43.3
507.6	-21.7	43.3
505.4	-45.3	43.3
501.6	-68.8	43.3
496.3	-91.9	43.3
489.6	-114.6	43.3
481.4	-136.9	43.3
471.9	-158.6	43.3
466.6	-169.2	43.3

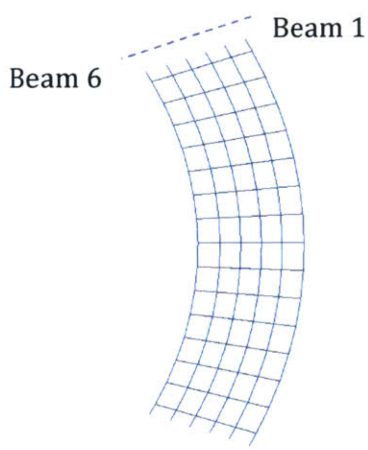
Beam 3		
x (ft)	y (ft)	z (ft)
450.0	167.2	32.6
455.2	157.0	32.6
464.7	136.1	32.6
473.0	114.7	32.6
479.8	92.7	32.6
485.2	70.4	32.6
489.1	47.7	32.6
491.6	24.9	32.6
492.5	1.9	32.6
491.8	-21.1	32.6
489.6	-43.9	32.6
486.0	-66.6	32.6
480.8	-89.0	32.6
474.2	-111.0	32.6
466.2	-132.6	32.6
456.9	-153.6	32.6
451.7	-163.8	32.6

Beam 4		
x (ft)	y (ft)	z (ft)
435.2	161.7	21.9
440.3	151.9	21.9
449.6	131.7	21.9
457.6	110.9	21.9
464.3	89.7	21.9
469.6	68.1	21.9
473.4	46.2	21.9
475.8	24.1	21.9
476.7	1.9	21.9
476.0	-20.4	21.9
473.9	-42.5	21.9
470.3	-64.5	21.9
465.3	-86.1	21.9
458.8	-107.4	21.9
451.0	-128.2	21.9
441.9	-148.5	21.9
436.9	-158.4	21.9

Beam 5		
x (ft)	y (ft)	z (ft)
420.4	156.2	11.3
425.4	146.7	11.3
434.4	127.2	11.3
442.3	107.2	11.3
448.8	86.7	11.3
454.0	65.8	11.3
457.7	44.7	11.3
460.0	23.3	11.3
460.9	1.8	11.3
460.3	-19.7	11.3
458.2	-41.1	11.3
454.7	-62.3	11.3
449.8	-83.3	11.3
443.5	-103.8	11.3
435.8	-123.9	11.3
427.0	-143.5	11.3
422.1	-153.1	11.3

Beam 6		
x (ft)	y (ft)	z (ft)
404.7	150.4	0
409.5	141.2	0
418.4	122.5	0
426.0	103.3	0
432.3	83.5	0
437.4	63.4	0
441.0	43.0	0
443.3	22.4	0
444.1	1.7	0
443.5	-19.0	0
441.5	-39.6	0
438.1	-60.1	0
433.3	-80.2	0
427.1	-100.0	0
419.7	-119.3	0
411.1	-138.2	0
406.3	-147.3	0

Column Base		
x (ft)	y (ft)	z (ft)
-	-	-
447.7	154.4	0
457.2	133.9	0
465.3	112.8	0
472.1	91.2	0
477.4	69.2	0
481.3	47.0	0
483.7	24.5	0
484.6	1.9	0
483.9	-20.7	0
481.8	-43.2	0
478.1	-65.5	0
473.0	-87.6	0
466.5	-109.2	0
458.6	-130.4	0
449.4	-151.0	0
-	-	-



Straight Geometry:

Beam 1		
x (ft)	y (ft)	z (ft)
-12.3	443	54.55
0.0	443	54.55
24.5	443	54.55
49.0	443	54.55
73.5	443	54.55
98.0	443	54.55
122.5	443	54.55
147.0	443	54.55
171.5	443	54.55
196.0	443	54.55
220.5	443	54.55
245.0	443	54.55
269.5	443	54.55
294.0	443	54.55
318.5	443	54.55
343.0	443	54.55
355.3	443	54.55

Beam 2		
x (ft)	y (ft)	z (ft)
-12.3	362.13	0
0.0	362.13	0
24.5	362.13	0
49.0	362.13	0
73.5	362.13	0
98.0	362.13	0
122.5	362.13	0
147.0	362.13	0
171.5	362.13	0
196.0	362.13	0
220.5	362.13	0
245.0	362.13	0
269.5	362.13	0
294.0	362.13	0
318.5	362.13	0
343.0	362.13	0
355.3	362.13	0

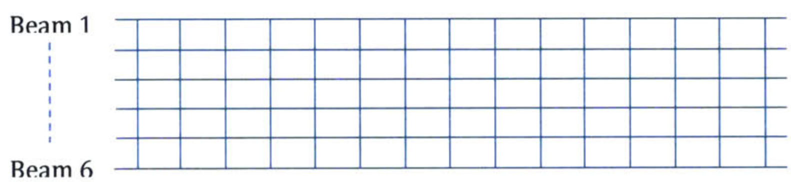
Beam 3		
x (ft)	y (ft)	z (ft)
-12.3	378.9	11.3
0.0	378.9	11.3
24.5	378.9	11.3
49.0	378.9	11.3
73.5	378.9	11.3
98.0	378.9	11.3
122.5	378.9	11.3
147.0	378.9	11.3
171.5	378.9	11.3
196.0	378.9	11.3
220.5	378.9	11.3
245.0	378.9	11.3
269.5	378.9	11.3
294.0	378.9	11.3
318.5	378.9	11.3
343.0	378.9	11.3
355.3	378.9	11.3

Beam 4		
x (ft)	y (ft)	z (ft)
-12.3	426.3	43.3
0.0	426.3	43.3
24.5	426.3	43.3
49.0	426.3	43.3
73.5	426.3	43.3
98.0	426.3	43.3
122.5	426.3	43.3
147.0	426.3	43.3
171.5	426.3	43.3
196.0	426.3	43.3
220.5	426.3	43.3
245.0	426.3	43.3
269.5	426.3	43.3
294.0	426.3	43.3
318.5	426.3	43.3
343.0	426.3	43.3
355.3	426.3	43.3

Beam 5		
x (ft)	y (ft)	z (ft)
-12.3	394.7	21.9
0.0	394.7	21.9
24.5	394.7	21.9
49.0	394.7	21.9
73.5	394.7	21.9
98.0	394.7	21.9
122.5	394.7	21.9
147.0	394.7	21.9
171.5	394.7	21.9
196.0	394.7	21.9
220.5	394.7	21.9
245.0	394.7	21.9
269.5	394.7	21.9
294.0	394.7	21.9
318.5	394.7	21.9
343.0	394.7	21.9
355.3	394.7	21.9

Beam 6		
x (ft)	y (ft)	z (ft)
-12.2	410.5	32.6
0.0	410.5	32.6
24.5	410.5	32.6
49.0	410.5	32.6
73.5	410.5	32.6
98.0	410.5	32.6
122.5	410.5	32.6
147.0	410.5	32.6
171.5	410.5	32.6
196.0	410.5	32.6
220.5	410.5	32.6
245.0	410.5	32.6
269.5	410.5	32.6
294.0	410.5	32.6
318.5	410.5	32.6
343.0	410.5	32.6
355.3	410.5	32.6

Column Base		
x (ft)	y (ft)	z (ft)
-	-	-
0.0	402.6	0
24.5	402.6	0
49.0	402.6	0
73.5	402.6	0
98.0	402.6	0
122.5	402.6	0
147.0	402.6	0
171.5	402.6	0
196.0	402.6	0
220.5	402.6	0
245.0	402.6	0
269.5	402.6	0
294.0	402.6	0
318.5	402.6	0
343.0	402.6	0
-	-	-



Appendix B

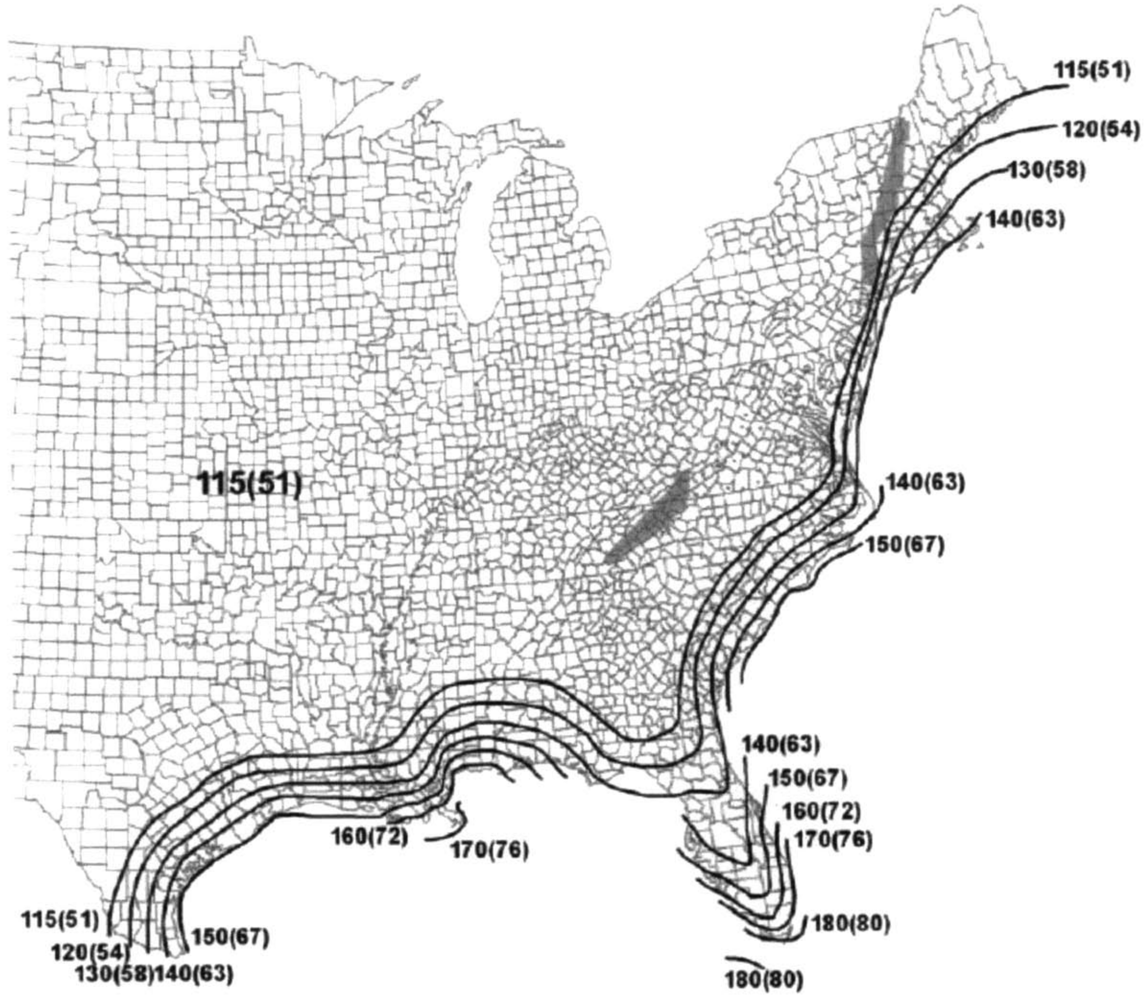
Study Summary		Beam 1		Beam 2	
Case Study	Connection Type	Section	Length (ft)	Section	Length (ft)
Ellipse 1	Welded	W18X55	367.4	W18X71	354.5
Ellipse 1	Bolted	W18X60	367.4	W18X76	354.5
Ellipse 2	Welded	W18X50	367.6	W18X76	355.7
Ellipse 2	Bolted	W18X65	367.6	W18X76	355.7
Straight	Welded	W18X50	367.6	W18X76	367.6
Straight	Bolted	W18X60	367.6	W18X76	367.6

Study Summary		Beam 3		Beam 4	
Case Study	Connection Type	Section	Length (ft)	Section	Length (ft)
Ellipse 1	Welded	W18X76	342.4	W18X76	330.2
Ellipse 1	Bolted	W18X76	342.4	W18X76	330.2
Ellipse 2	Welded	W18X76	344.6	W18X76	333.5
Ellipse 2	Bolted	W18X76	344.6	W18X71	333.5
Straight	Welded	W18X76	367.6	W18X76	367.6
Straight	Bolted	W18X76	367.6	W18X76	367.6

Study Summary		Beam 5		Beam 6	
Case Study	Connection Type	Section	Length (ft)	Section	Length (ft)
Ellipse 1	Welded	W18X55	318	W18X40	305.2
Ellipse 1	Bolted	W18X76	318	W18X50	305.2
Ellipse 2	Welded	W18X55	322.5	W18X40	310.7
Ellipse 2	Bolted	W18X71	322.5	W18X50	310.7
Straight	Welded	W18X76	367.6	W18X50	367.6
Straight	Bolted	W18X76	367.6	W18X60	367.6

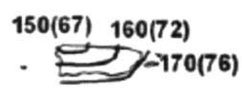
Study Summary		Stringer				
Case Study	Connection Type	Section	Length (ft)	Total Wt (tons)	Seats Per Segment	Steel Wt per Seat (lbs/seat)
Ellipse 1	Welded	W30X116	1463	148	6156	48.1
Ellipse 1	Bolted	W30X116	1463	154.6	6156	50.2
Ellipse 2	Welded	W30X116	1463	148.4	6095	48.7
Ellipse 2	Bolted	W33X118	1463	155.9	6095	51.2
Straight	Welded	W33X118	1463	160.6	6720	47.8
Straight	Bolted	W33X118	1463	164.2	6720	48.9

Appendix C



Special Wind Region

Location	Vmph	(m/s)
Guam	195	(87)
Virgin Islands	165	(74)
American Samoa	160	(72)
Hawaii – Special Wind Region Statewide	130	(58)



Puerto Rico

Figure 26.5-1A (*Continued*)

Appendix D

STATION	STATION_NAME	YEAR	Max WESD (0.1mm)	Max WESD, x (in)	Ln(x)
GHCND:USW00014739	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1953	645	2.54	0.93
GHCND:USW00014740	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1954	323	1.27	0.24
GHCND:USW00014741	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1955	183	0.72	-0.33
GHCND:USW00014742	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1956	587	2.31	0.84
GHCND:USW00014743	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1957	389	1.53	0.43
GHCND:USW00014744	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1958	627	2.47	0.90
GHCND:USW00014745	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1959	381	1.50	0.41
GHCND:USW00014746	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1960	483	1.90	0.64
GHCND:USW00014747	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1961	960	3.78	1.33
GHCND:USW00014748	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1962	635	2.50	0.92
GHCND:USW00014749	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1963	323	1.27	0.24
GHCND:USW00014750	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1964	457	1.80	0.59
GHCND:USW00014751	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1965	318	1.25	0.22
GHCND:USW00014752	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1966	607	2.39	0.87
GHCND:USW00014753	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1967	470	1.85	0.62
GHCND:USW00014754	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1968	0	0.00	0.00
GHCND:USW00014755	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1969	843	3.32	1.20

GHCND:USW00014756	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1970	457	1.80	0.59
GHCND:USW00014757	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1971	279	1.10	0.09
GHCND:USW00014758	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1972	635	2.50	0.92
GHCND:USW00014759	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1974	457	1.80	0.59
GHCND:USW00014760	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1975	813	3.20	1.16
GHCND:USW00014761	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1976	0	0.00	0.00
GHCND:USW00014762	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1977	1219	4.80	1.57
GHCND:USW00014763	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1978	1143	4.50	1.50
GHCND:USW00014764	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1979	381	1.50	0.41
GHCND:USW00014765	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1980	127	0.50	-0.69
GHCND:USW00014766	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1981	457	1.80	0.59
GHCND:USW00014767	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1982	533	2.10	0.74
GHCND:USW00014768	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1983	610	2.40	0.88
GHCND:USW00014769	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1984	432	1.70	0.53
GHCND:USW00014770	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1985	229	0.90	-0.10
GHCND:USW00014771	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1986	178	0.70	-0.36
GHCND:USW00014772	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1987	0	0.00	0.00
GHCND:USW00014773	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1988	483	1.90	0.64

GHCND:USW00014774	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1989	3810	15.0	2.71
GHCND:USW00014775	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1990	305	1.20	0.18
GHCND:USW00014776	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1991	279	1.10	0.09
GHCND:USW00014777	BOSTON LOGAN INTERNATIONAL AIRPORT MA US	1992	203	0.80	-0.22

Appendix E

Upper Outer Ring		
x (ft)	y (ft)	z (ft)
0.0	458.0	20.0
25.3	457.5	20.0
50.7	456.0	20.0
75.8	453.5	20.0
100.9	449.9	20.0
125.8	445.4	20.0
150.5	439.8	20.0
174.9	433.2	20.0
199.1	425.7	20.0
222.9	417.1	20.0
246.3	407.5	20.0
269.2	396.9	20.0
291.7	385.3	20.0
313.6	372.7	20.0
334.8	359.1	20.0
355.5	344.6	20.0
375.4	329.0	20.0
394.5	312.6	20.0
412.7	295.1	20.0
430.0	276.8	20.0
446.4	257.6	20.0
461.6	237.5	20.0
475.7	216.6	20.0
488.5	194.9	20.0
500.2	172.5	20.0
510.4	149.5	20.0
519.2	125.9	20.0
526.5	101.7	20.0
532.2	77.2	20.0
536.4	52.4	20.0
539.1	27.3	20.0
540.0	0.0	20.0

Outer Ring		
x (ft)	y (ft)	z (ft)
0.0	443.0	0.0
24.5	442.5	0.0
49.0	441.1	0.0
73.3	438.7	0.0
97.6	435.3	0.0
121.7	430.9	0.0
145.6	425.6	0.0
169.3	419.3	0.0
192.7	412.1	0.0
215.8	403.9	0.0
238.5	394.7	0.0
260.8	384.5	0.0
282.6	373.3	0.0
303.9	361.2	0.0
324.6	348.2	0.0
344.7	334.1	0.0
364.1	319.2	0.0
382.7	303.2	0.0
400.5	286.4	0.0
417.4	268.7	0.0
433.4	250.1	0.0
448.3	230.6	0.0
462.0	210.4	0.0
474.6	189.3	0.0
486.0	167.6	0.0
496.0	145.3	0.0
504.6	122.3	0.0
511.8	98.9	0.0
517.4	75.0	0.0
521.5	50.9	0.0
524.1	26.5	0.0
525.0	0.0	0.0

Middle Ring		
x (ft)	y (ft)	z (ft)
0.0	403.0	0
22.3	402.6	0
44.6	401.3	0
66.7	399.2	0
88.8	396.2	0
110.8	392.4	0
132.7	387.8	0
154.3	382.2	0
175.8	375.8	0
196.9	368.6	0
217.8	360.4	0
238.3	351.4	0
258.5	341.4	0
278.1	330.6	0
297.3	318.9	0
316.0	306.3	0
334.0	292.8	0
351.3	278.4	0
368.0	263.1	0
383.8	247.0	0
398.8	230.1	0
412.7	212.3	0
425.6	193.8	0
437.4	174.5	0
448.2	154.6	0
457.6	134.0	0
465.7	112.9	0
472.5	91.3	0
477.8	69.3	0
481.7	47.0	0
484.2	24.5	0
485.0	0.0	0

Inner Ring		
x (ft)	y (ft)	z (ft)
0.0	363.0	0
20.1	362.6	0
40.2	361.6	0
60.1	359.8	0
80.1	357.2	0
100.0	353.9	0
119.7	349.9	0
139.3	345.1	0
158.8	339.6	0
178.1	333.3	0
197.1	326.2	0
215.9	318.3	0
234.3	309.6	0
252.4	300.0	0
270.0	289.6	0
287.3	278.5	0
303.9	266.4	0
320.0	253.6	0
335.4	239.9	0
350.1	225.4	0
364.1	210.1	0
377.2	194.0	0
389.2	177.2	0
400.3	159.7	0
410.4	141.5	0
419.2	122.8	0
426.9	103.5	0
433.3	83.7	0
438.2	63.6	0
441.9	43.1	0
444.2	22.5	0
445.0	0.0	0